SURROUNDING ROCK STABILITY CALCULATION OF ADIT NO.3

[UPPER TRISHULI-1 HEP (216MW)]

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Doosan Heavy Industries & Construction

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Project

Upper Trishuli-1 HEP (216MW)

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1 General

1.1 Background

Upper Trishuli-1 HEP is constructed to utilize the hydro potential of Trishuli River near Dunche and Haku. The construction site is situated in a rugged mountain between Tibet in China and the Himalaya Mountain, about 70 km away from capital of Nepal, Kathmandu, towards the northwest. The project is a run-of-river development scheme. The headwork site is at about 280m downstream from the confluence of Bhotekoshi and Trishuli rivers. The powerhouse site is 500m upstream from Mailun Dobhan where Haku VDC is placed. The installed capacity of Upper Trishuli-1 HEP is 216 MW. It is a year-round power plant capable of generating about 1,533.4 GWh electricity yearly on average.

The Adit No.3 is located on the left bank of headrace tunnel, and the headrace tunnel chainage at intersection area is H+6562.747m. The cross section of headrace tunnel is horseshoe shape and 4.2m×5.6m size. The elevation of the inlet is EL. 1194.28m, and the elevation of outlet is EL.1212.36m. The length of Adit No.3 is 313.126m and longitudinal slope is 5.8%.

According to the Contractor's interpretation and preliminary geological survey data, along the tunnel axis, there are interbedded mica schist and quartz schist are distributed and they are classified as Type Ⅳ,Ⅲ,Ⅱ as per Q-method from inlet to outlet, the buried depth from inlet to outlet is about 50m to 300m, averaged 200m approximately. It is speculated that the whole tunnel is mostly dry, individual joints surface is wet. No potential continuous sliding surface is found on the tunnel slope.

The longitudinal profile of Adit No.3 is shown in Figure 1.1. The preliminary excavation cross section and preliminary support pattern are shown in Figure 1.2~1.5.

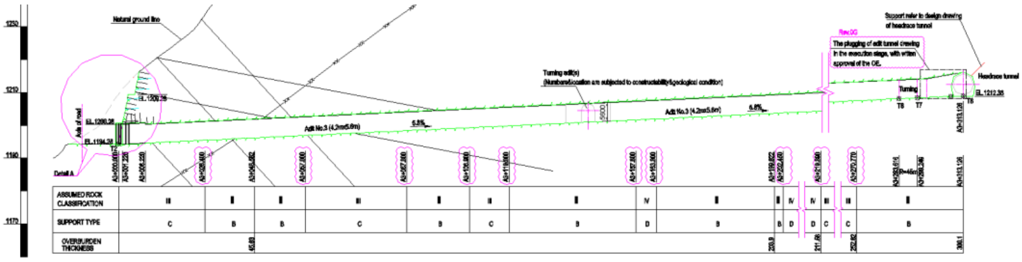


Figure 1.1 Longitudinal Profile Section of Adit No.3

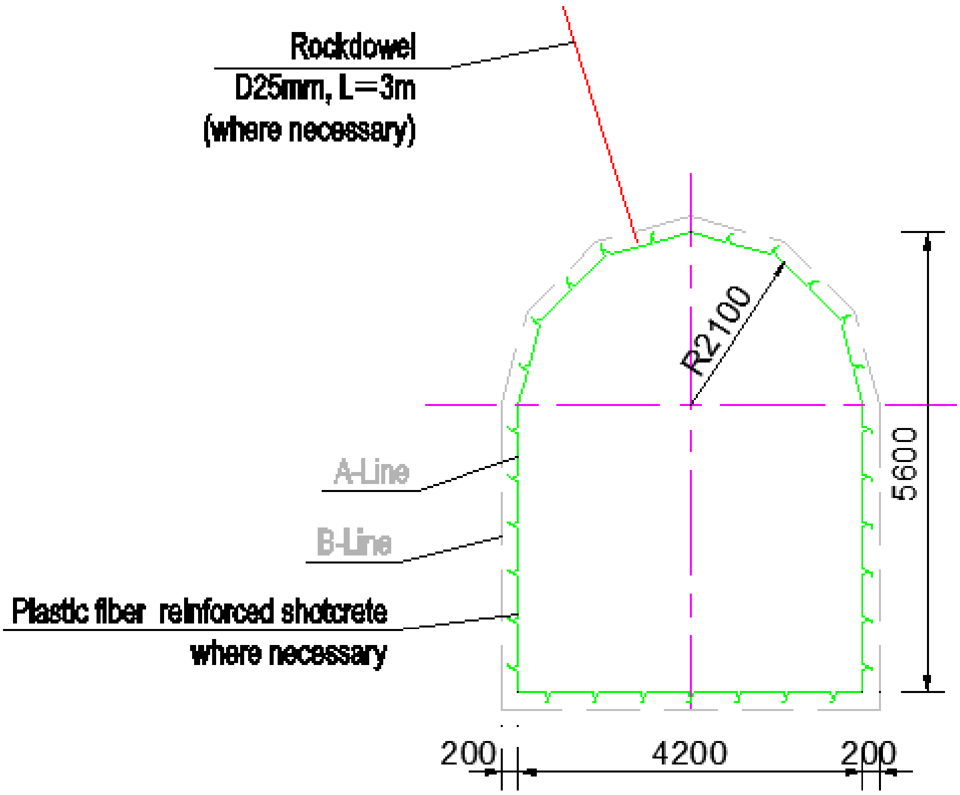


Figure 1.2 Typical section of adit No.3 (rock class I)

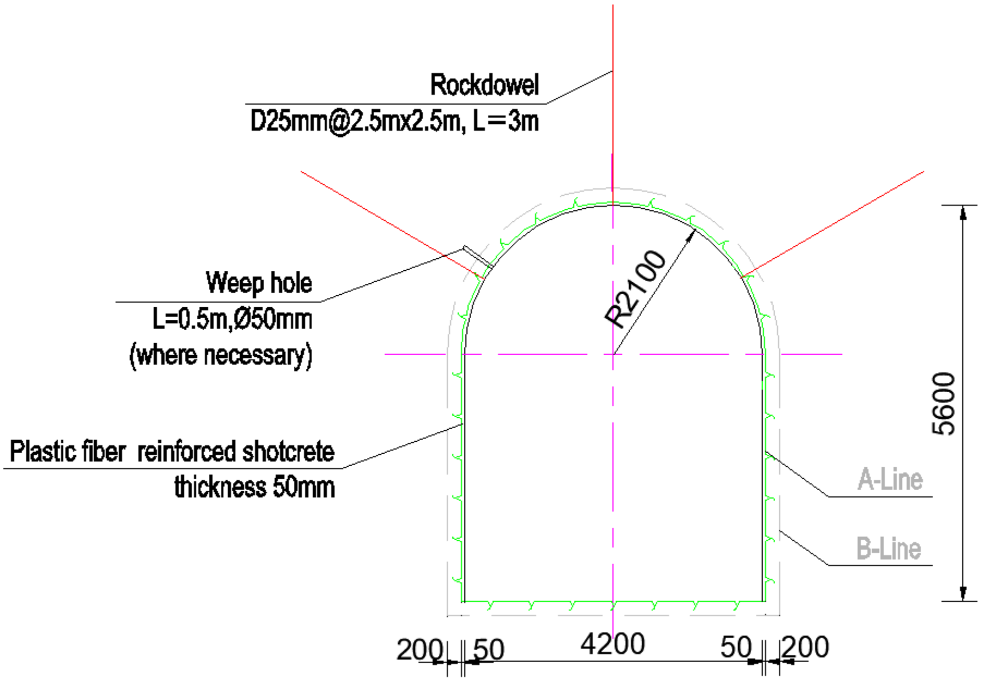


Figure 1.3 Typical section of adit No.3 (rock class II)

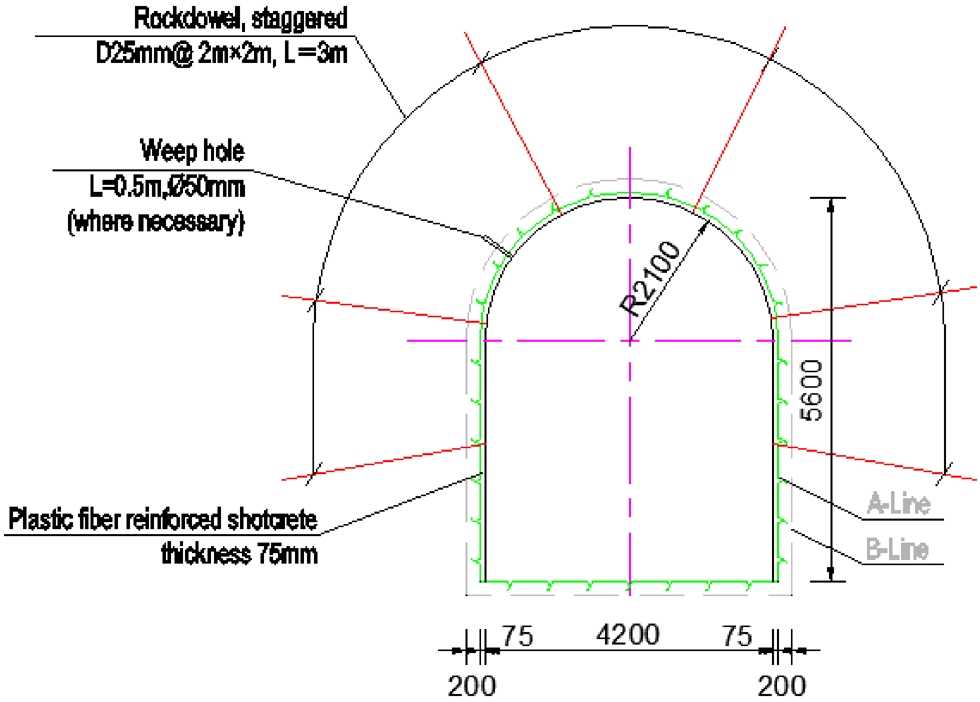


Figure 1.4 Typical section of adit No.3 (rock class III)

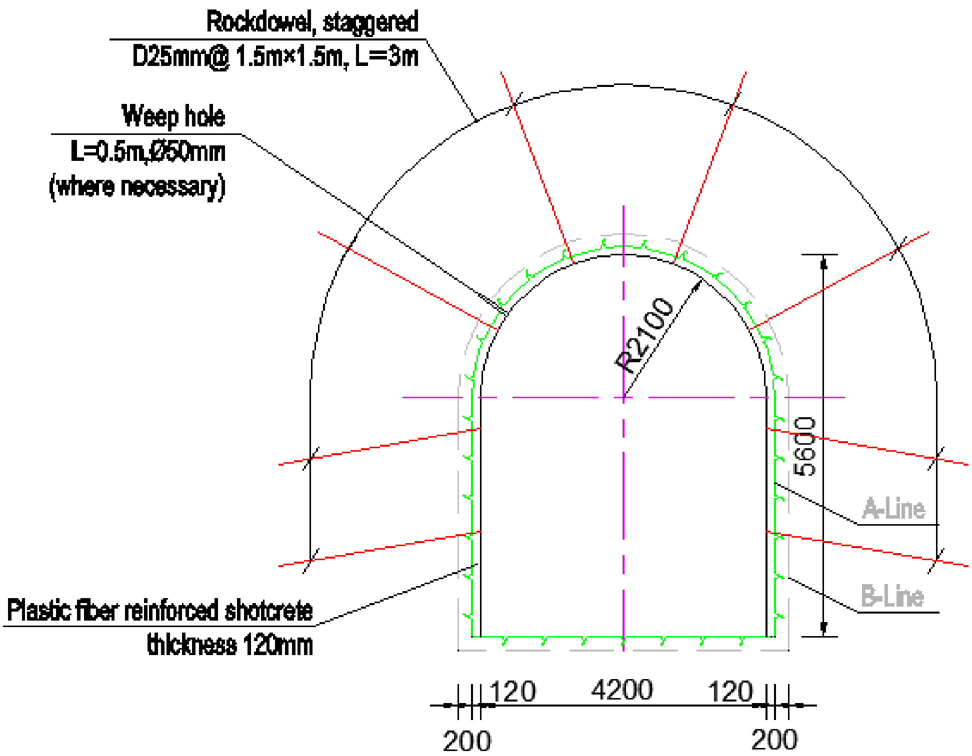


Figure 1.5 Typical section of adit No.3 (rock class IV)

1.2 Design Objective

This calculation only includes rock class I~IV, and the rock class V shall be considered separately according to the actual situation.

This calculation is provided for detailed design stage and aim to confirm the objectives which are as following:

(1) to generally confirm the stability of the tunnel during construction;

(2) to confirm (or adjust) the previously suggested support measures in the construction drawings.

1.3 Reference

(1) Contract documents, Employer’s Requirement;

(2) Tunnels and shafts in Rock (EM 1110-2-2901);

(3) Rock Foundations (EM 1110-1-2908);

(4) Building Code Requirements for Structural Concrete and Commentary (ACI 318)

(5) Vlachopoulos, N. and Diederichs, M.S. (2009). Improved longitudinal displacement profiles for convergence-confinement analysis of deep tunnels. Rock Mechanics and Rock Engineering, 42(2), 131-146;

(6) The 2008 Kersten Lecture Integration of geotechnical and structural design in tunneling; To be presented at the opening keynote address, by Dr Evert Hoek, at the University of Minnesota 56th Annual Geotechnical Engineering Conference to be held in Minneapolis on 29 February 2008;

(7) Handbook Using the Q-system, Rock mass classification and support design;

(8) Paper: Rock-Support Interaction analysis for tunnels in weak rock masses;

(9) https://www.rocscience.com/help/rs2/turorials/rs2 _tunnel lining design.htm;

(10) Drawing: Excavation and initial support drawing of adit No.3 (Drawing No. UT1-C-150-CVL-DG-43004).

2 Theory of computation

2.1 Software

Software Phase 2D will be applied in the calculation. Phase2 is a powerful 2D finite element program for soil and rock applications. Phase2 can be used for a wide range of engineering projects including excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation, and dynamic analysis capabilities.

Software UNWEDGE is an interactive software for the stability analysis of three-dimensional wedge formed by structural discontinuity and underground excavation, which is used to analyze the underground excavation problem with discontinuous structural plane in rock mass. UNWEDGE calculates the safety factor of potentially unstable wedge, and can analyze the influence of support system on wedge stability.

2.2 Modelling Conditions, Material Law, Constitutive Model

The modelling conditions are plane strain. The material law is elasto-plastic combined with strain-softening (for rock). The rock mass is modelled as a continuum, homogeneous and isotropic. The failure criteria is the Mohr-Coulomb failure criteria. What needs to be emphasized is that such continuum model with homogeneous and isotropic material is a model, only, and does not necessarily reflect reality. In reality, the displacements occur in blocky rock masses always along discontinuities (joints, foliation etc.), which leads to loosening of the rock mass. The determined plastic zone in the continuums model represents an area of the rock mass where rock blocks were displaced to each other (= loosening of rock mass). In reality, it does not represent an area where the material was subject to plastic flow, as in soil.

2.3 Calculation Methodology

Convergence-confinement analysis for tunnelling is a standard approach for preliminary analysis of anticipated wall deformation and support design in squeezing ground.

Convergence-confinement analysis is a widely used tool for preliminary assessment of squeezing potential and support requirements for circular tunnels in variety of geological conditions and stress states. An analytical plasticity solution such as that developed by Carranza-Torres and Fairhurst (2000) is applied to a circular opening in an isotropic stress field. An internal pressure, initially equal to the in-situ stress is applied on the inside of the excavation boundary. The pressure is incrementally relaxed until the excavation boundary condition is that of zero normal stress. The extent of plastic yielding and thereby, the boundary deformation is calculated at each stage of the process. The result is a continuous representation of the deformation-internal pressure relationship for the tunnel given a particular material

strength, deformability, dilation and stress state.

The internal press is, of course not a representation of reality but rather a surrogate for the effect of the gradual reduction of the radial resistance provided by the initially present tunnel core (material inside the tunnel boundary) transitioning to an exposed boundary with a progressively distant tunnel face and ultimately a long open tunnel with plane strain conditions. The internal pressure that is coupled with a given boundary displacement is a measure of the amount of support resistance required to prevent further displacement at that point in progressive tunnelling model.

Elastic deformation of the rock mass starts about two diameters ahead of the advancing face and reaches its maximum value at about two diameters behind the face. At the face position about one third of the total radial closure of the tunnel has already occurred and the tunnel face deforms inwards as illustrated in Fig. 2.1. Whether or not these deformations induce stability problems in the tunnel depends upon the ratio of rock mass strength to the in-site stress level. The tunnel displacement profiles for the roof and the invert of a tunnel are shown schematically in the Fig. 2.2.

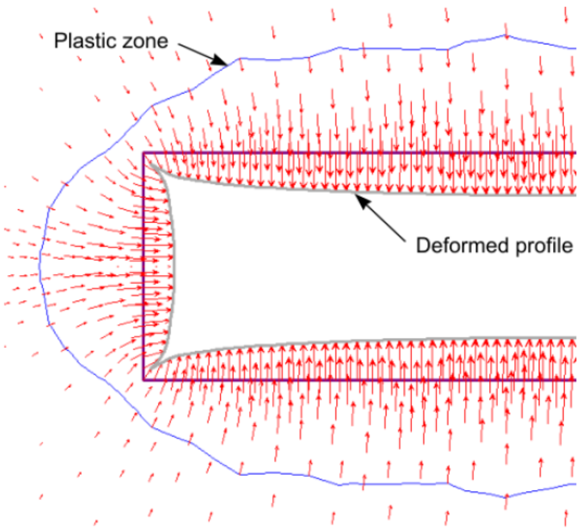


Figure 2.1 The shape of the deformed tunnel profile and displacement vectors

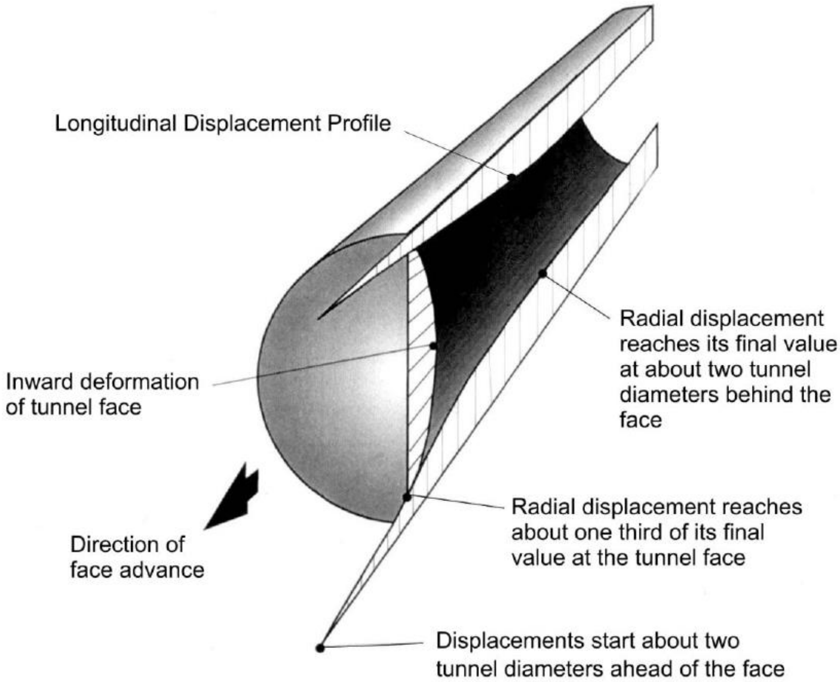


Figure 2.2 pattern of elastic deformation in the rock mass surrounding an advancing tunnel

The equations can be found in the Kersten Lecture, appendix 1(Hoek et.al.,2008)

According to the formula listed in geological rock classification and documentation "Integration of geotechnical and structural design in tunneling (Hoek, E., Carranza-Torres, C., Diederichs, M.S. and Corkum, 2008)". The calculation formula of supporting time is as follow:

$$\frac{u_0}{u_{max}} = \frac{1}{3} e^{-0.15P_r}$$

$$P_r = R_p / R_t$$

$$d_t = \frac{X}{R_t}$$

$$\frac{u}{u_{max}} = \begin{cases} \frac{u_0}{u_{max}} e^{d_t} & \text{when} X < 0 \\ 1-(1-\frac{u_0}{u_{max}}) e^{\frac{-3d_t}{2P_r}} & \text{when} X > 0 \end{cases}$$

where,

R_p —The maximum plastic zone depth, m;

u_0 —The radial displacement of tunnel face, m;

R_t —Tunnel radius, m;

X —Distance from the initial support to the tunnel face, m;

u_{max} —The maximum radial displacement, m.

With these analytical equations it is possible to estimate the amount of displacements at the specified location of support installation, if the plastic radius and displacement far from the tunnel face are known. Therefore, to get the displacements and the radius of the plastic zone far away from the tunnel face (where the internal pressure is zero). It is necessary to calculate numerically with Phase 2 the tunnel without support, first.

With the determined displacement profile along the tunnel and a given location of support installation, the modeler selects the corresponding deformation.

In the next step the tunnel advance is modelled in Phase 2D by reducing the internal pressure in the future tunnel until the corresponding deformation is reached, when support is installed. The modeler installs now the support in the model. The internal pressure is then reduced to zero by factoring the field stress zero. Further numerical calculation leads to loading of the support.

2.4 Maximum Unsupported Span

The maximum unsupported span is calculated by the following formula as per EM 2901:

X=2 ESR Q^0.4

Where, X is maximum unsupported span, ESR is excavation support ratio, taken as 1.3. Q values and calculated and applied maximum unsupported spans are shown Table 2.1 :

Table 2.1 Maximum unsupported span of tunnel excavation

Rock class	Q Value	Calculated maximum unsupported span(m)	Maximum unsupported span for application, X(m)
I	40	11.4	11
II	25	9.4	9
III	7	5.7	5
IV	2.5	3.8	3

2.5 Ground Characteristic Curve

Define the relationship between stress release ratio λ_i and the tunnel deformation u_i for an advancing tunnel in stress field. A plot of u_i versus λ_i , the curve is based on the assumption that the rock at the tunnel face provides an initial support pressure equal to the in-situ stress. As the tunnel face advances and the face moves away from the section under consideration, the support pressure gradually decreases until it reaches zero at some distance behind the face. The characteristic curve of class I~IV are shown as below.

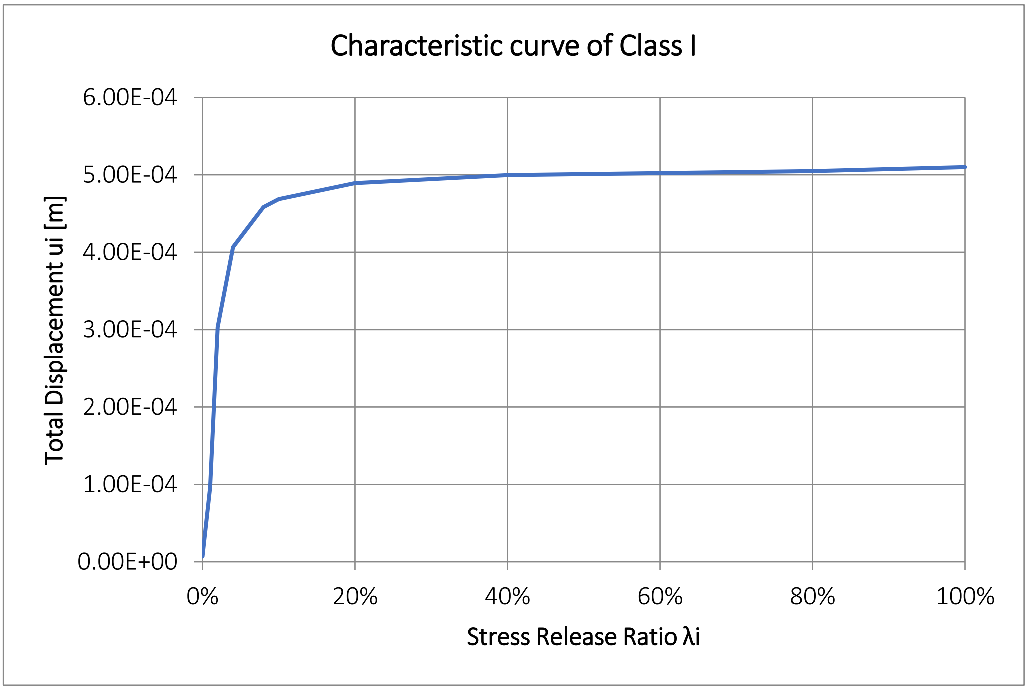


Figure 2.3 Characteristic curve of class I

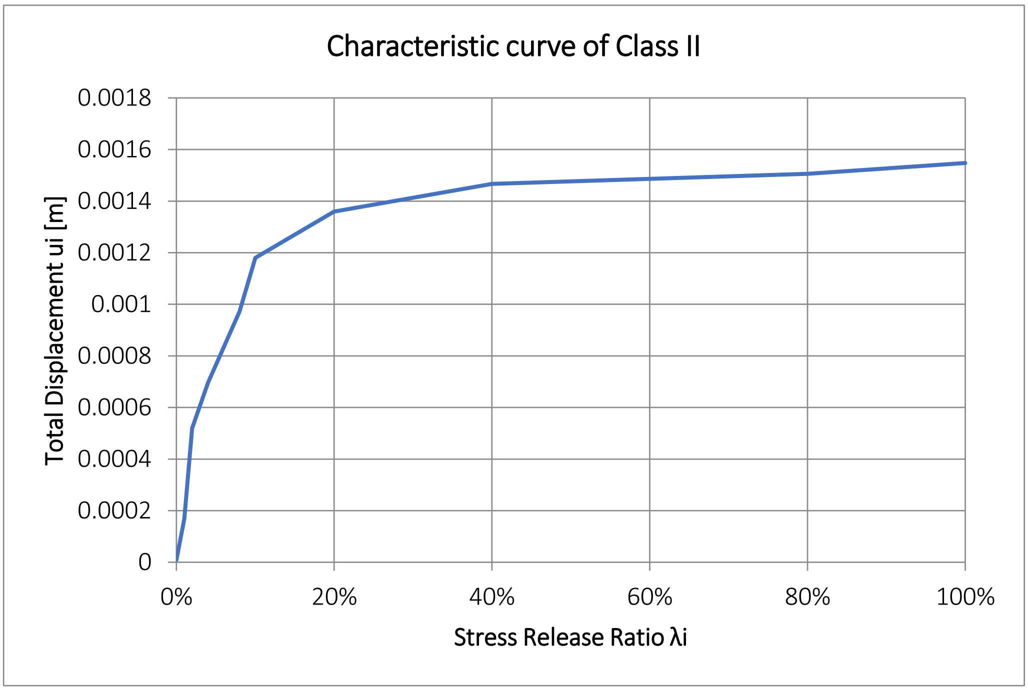


Figure 2.4 Characteristic curve of class II

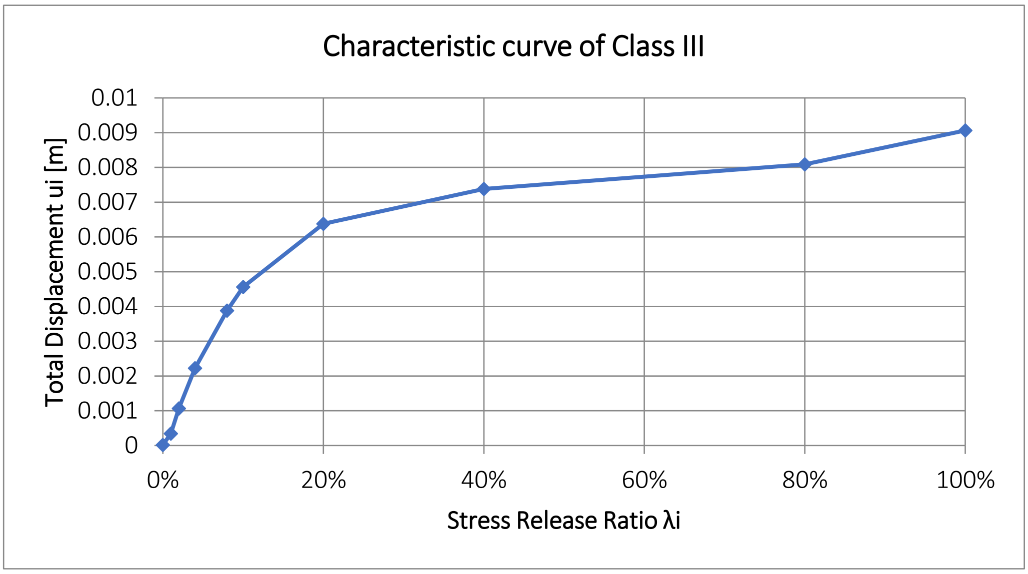


Figure 2.5 Characteristic curve of class III

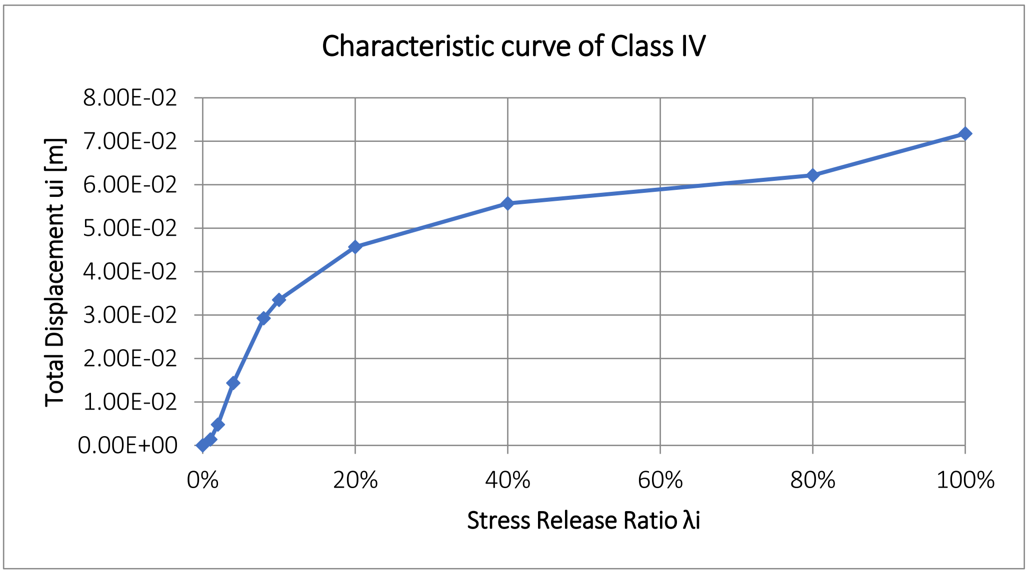


Figure 2.6 Characteristic curve of class IV

2.6 Longitudinal displacement profile (LDP) for tunnel

The Longitudinal Displacement Profile (LDP) is required in order to establish the relative position of the tunnel face and the section under consideration.

According to the LDP equations, presented above, the LDP curve of every rock mass class are shown in the table below.

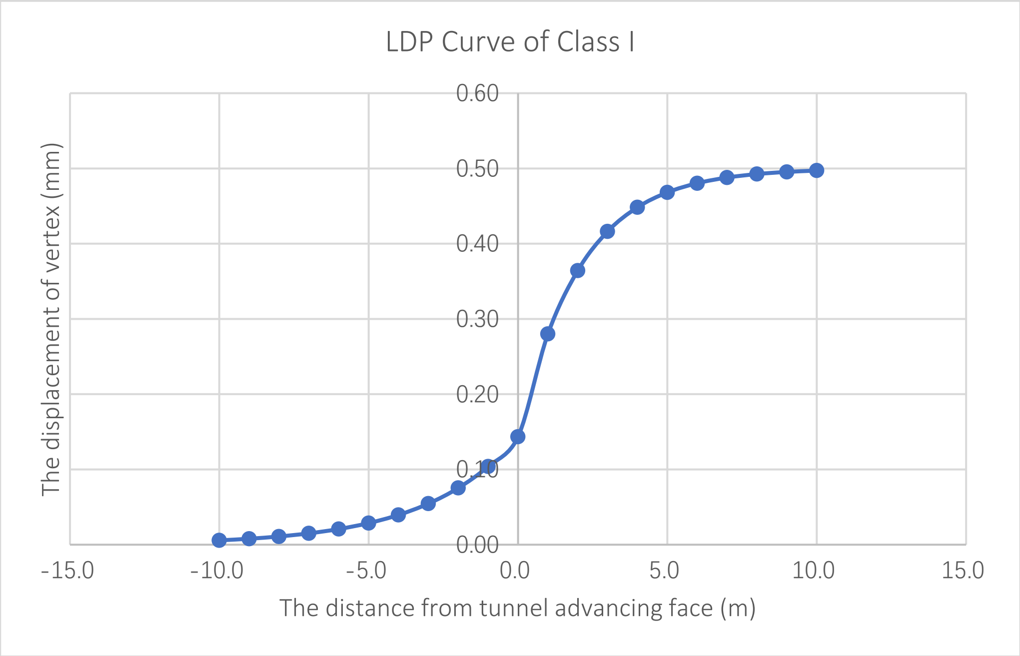


Figure 2.7 Longitudinal Displacement Profile curve of class I

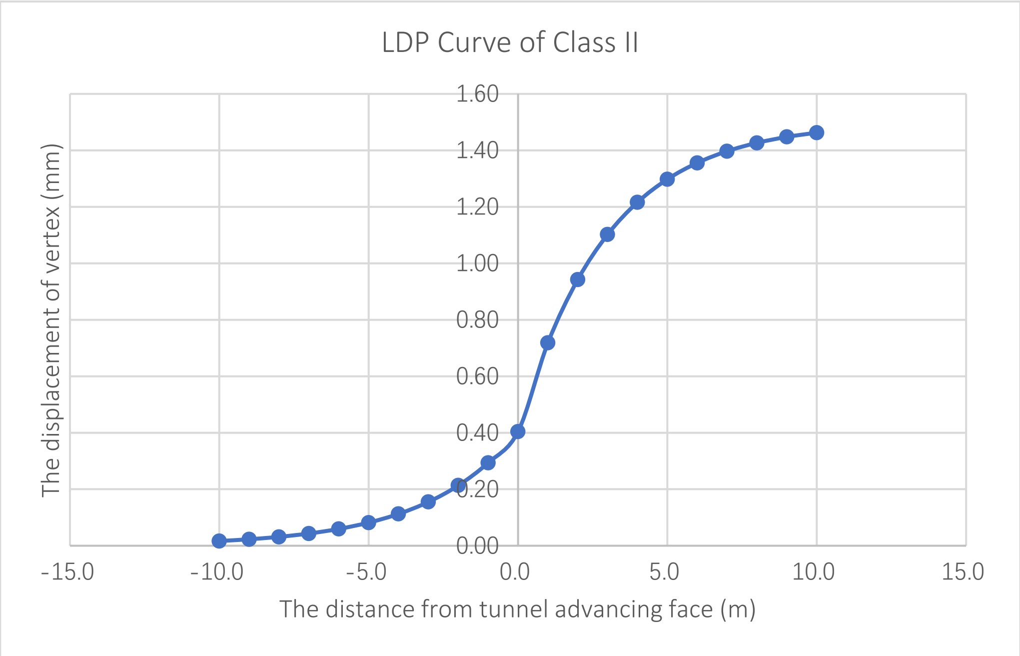


Figure 2.8 Longitudinal Displacement Profile curve of class II

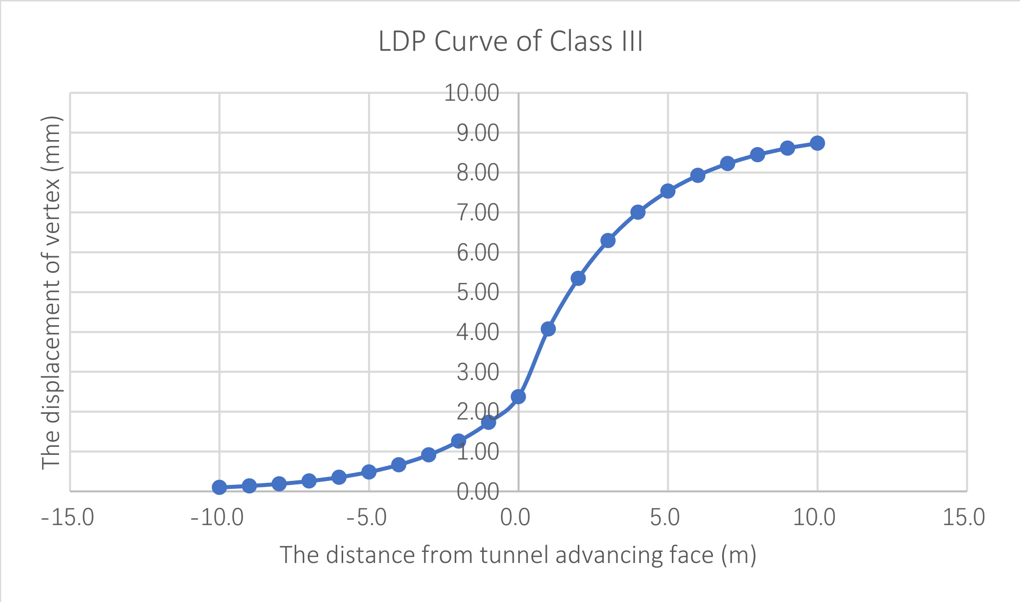


Figure 2.9 Longitudinal Displacement Profile curve of class III

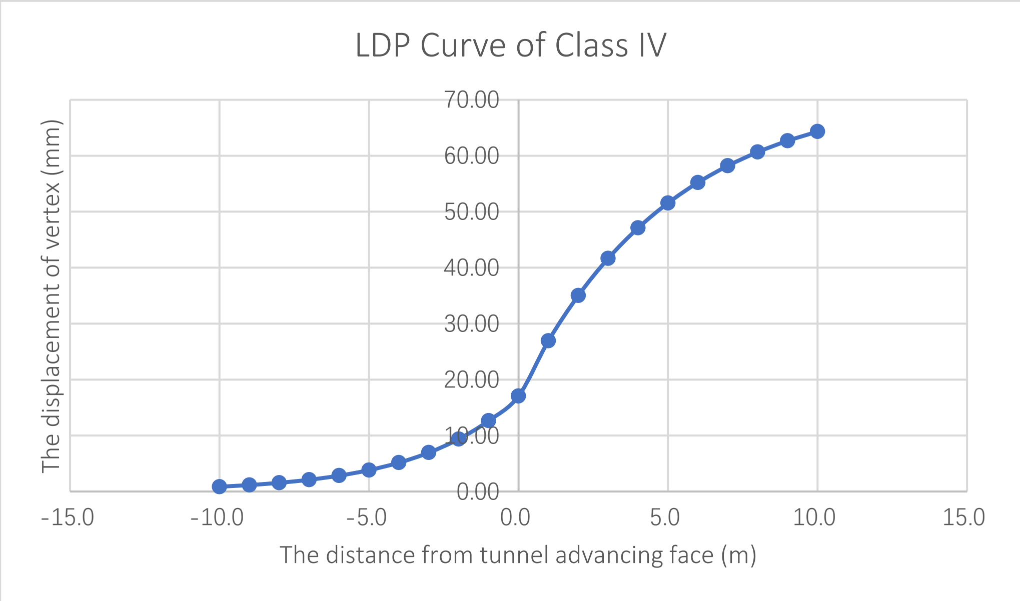


Figure 2.10 Longitudinal Displacement Profile curve of class IV

According to Characteristic Curve and LDP Curve of class I~IV, it is determined to adopt the relaxation of stress at the supporting time shown as below.

Table 2.2 Relaxation of rock mass

Rock Class	Supporting time X (m)	Plastic zone radius R_p (m)	Vertex displacement U_{max} (mm)	$\frac{u}{u_{max}}$ Closure ratio	Stress Relaxation Percentage (%)
I	11	3.62	0.5	0.99	***95***
II	9	4.44	1.5	0.97	***95***
III	5	5.15	9.1	0.83	***92***
IV	3	7.54	71.8	0.58	***90***

Relaxation refer to that (100-Relaxation)% of surrounding rock stress is undertaken by rock and support, Relaxation % release during excavation. The stress relaxation percentage has already considered the margin of safety.

3 Properties of Materials

3.1 Rock Mass Properties

The mechanical parameters of surrounding rock are shown in Table 3.3.

Table 3.1 Mechanical parameter of surrounding rock

Class of surrounding rock	Unit weight	Uniaxial compressive strength (UCS) of intact rock	Elastic modulus Of rock mass	Poisson's ratio of rock mass	Rock mass/rock mass shear (shearing) strength				Uniaxial compressive strength of rock mass, (UCS)	Tensile Strength of rock mass
					Peak		Residual			
	γ (kN/m^3)	σ_c (MPa)	E_m (GPa)	μ	Φ_{peak} (°)	C_{peak} (MPa)	Φ_{res} (°)	C_{res} (MPa)	σ_{cm} (MPa)	σ_{tm} (MPa)
Ⅰ	27.0	60	30.0	0.20	50	5.0	40	3.33	27.5	2.75
Ⅱ	27.0	60	20.5	0.25	48	2.5	38	1.67	13.0	1.3
Ⅲ	26.5	60	10.0	0.25	45	1	36	0.66	4.8	0.48
IV	26.0	60	2.3	0.30	35	0.5	28	0.33	1.9	0.19

Note: 1) Residual strength parameters shall be reduced by 1/3 for the cohesion and 20% for the friction angle; 2) Dilation angle was estimated at 1/3 of the friction angle; 3) Tensile strength of rock mass is taken as 10% of uniaxial compressive strength of rock mass.

3.2 Shotcrete

a) Plastic fiber shotcrete: unit weight: 24.0kN/m³;

b) The Elastic moduli are calculated as formula E_c=4700*f_c'^0.5 (ACI 318, section 19.2.2 Modulus of elasticity).

Table 3.2 Shotcrete strength and elastic modulus

Compressive strength f'_c(MPa)	Elastic modulus E_c(MPa)	Poisson ratio
25	23500	0.2
30	25743	0.2

3.3 Rock Dowels

Rock dowel type: Cement mortar full length bonded dowel

1) Yield strength: 500MPa

2) Compressive/Tensile strength: 545MPa

3) Elastic modulus: 210,000MPa

4) Tensile strength of rock bolt steel: 500MPa/1.5=333MPa,

And the value 1.5 is safety factor.

Ultimate bearing capacity of rock dowel is calculated from formula below:

$$F=0.5\pi dL\tau_{ult}$$

Where,

d =Effective diameter of borehole;

L = Length of grouted portion of anchor;

t =Working bond strength;

τ_{ult}= The ultimate bond strength at failure, the ultimate bond stress is often taken as 1/10 of the uniaxial compressive strength of the rock or grout (whichever is less) (Littlejohn 1977) up to a maximum value of 4.2 MPa.

Table 3.3 The ultimate bearing capacity calculation of rock dowel

Rock classification	Unit	II	III	IV
UCS	MPa	60	60	60
Rock dowel diameter	mm	25	25	25
Rock dowel area	mm^2	491	491	491
Ultimate bond strength of rock dowel τ_{ult}	kPa	4200	4200	4200
Effective diameter of rock dowel borehole d	mm	50	50	50
Bond strength of unit length $F=0.5\pi d\tau_{ult}$	MN/m	0.33	$0.5\pi\times50\times4200/10^6=$ 0.33	0.33
Tensile strength of rock dowel steel	MPa	333	333	333
Tensile peak value of rock dowel	MN	0.164	$491\times333/10^6=$ 0.164	0.164

3.4 Lattice Girder and Reinforcing Bar

The mechanical parameters of reinforcing bar are shown as below.

Table 3.4 Mechanical parameter for reinforcing bar

Yield strength (MPa)	Tension strength(MPa)	Elastic modulus E_c(MPa)	Poisson ratio
500	545	210,000	0.3

3.5 Steel support

MB150 H-beam type steel rib is used. And the mechanical parameters of steel support are shown as below.

Table 3.5 Mechanical parameter for steel support

Yield strength (MPa)	Tension strength(MPa)	Elastic modulus E_c(MPa)	Poisson ratio
250	400	210,000	0.3

4 Initial Proposed Tunnel Supports

4.1 Tunnel Support of Excavation Class

The initial proposed supports for each rock type are shown as below.

Table 4.1 Initial proposed supports for each rock type

Item	Unit	Initial proposed supports			
		I	II	III	IV
Shotcrete thickness	mm	Spot	50	75	100
Shotcrete strength	MPa	21	21	21	21
Rock dowel length	m	3.0	3.0	3.0	3.0
Rock dowel spacing within each row	m	spot	spot	2.0	1.5
Rock dowel row spacing	m	spot	spot	2.0	1.5
Steel support/Lattice girder spacing	m				spot

4.2 Safety Factor

(1) The support safety factor for shotcrete and rock dowel are 1.2.

(2) The minimum safety factor for wedges analysis is 2.0.

5 Model Set-up, Boundary and Loading Conditions, Stress Initiation

For simplicity, the model will not extend to the ground surface, and the boundary of model is set 10 times tunnel width/height, but to apply corresponding stresses at the model's upper boundary.

For the case of gravitational loading, only, boundary and loading conditions to be set as shown below. This applies for tunnels in the UT1-Project that are oriented NNE-SSW (+/- 45°), as NE-SSW is generally orientation of the maximum horizontal stress σ_H in the Upper Himalaya, which is in the plane-strain Phase2-model the ("irrelevant") out-of-plane stress. Hence, initial stress conditions in the model can be calculated by gravitational loading, or by specifying the relative ground surface elevation.

The finite element model for rock class I, II, III, IV is shown as Figure 5.1, and the corresponding gravitational loading is shown in the Table 5.1.

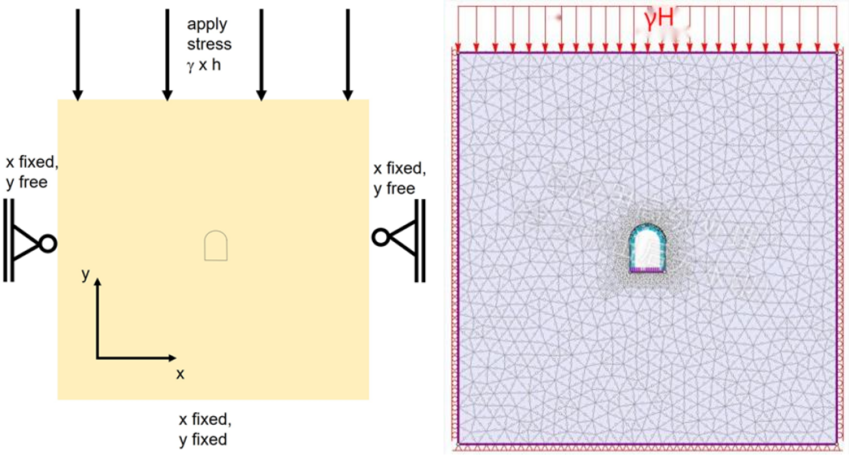


Figure 5.1 Finite element model and gravitational loading

Table 5.1 gravitational loading for surrounding rock

Rock class/ Location	Tunnel buried depth	Rock unit weight	Overburden gravitational loading	Application location of Max. buried depth
	m	kN/m³	MPa	
I	300	27	8.1	Refer to rock class II location
II	300	27	8.1	Whole tunnel
III	300	26.5	7.95	Whole tunnel
IV	210	26	5.46	Site expose

6 Tunnel Modelling and Results

6.1 Rock Class I

For surrounding rock class I without support, adopting the parameters listed in Section 3, and the calculated displacement and plastic zone are shown as below.

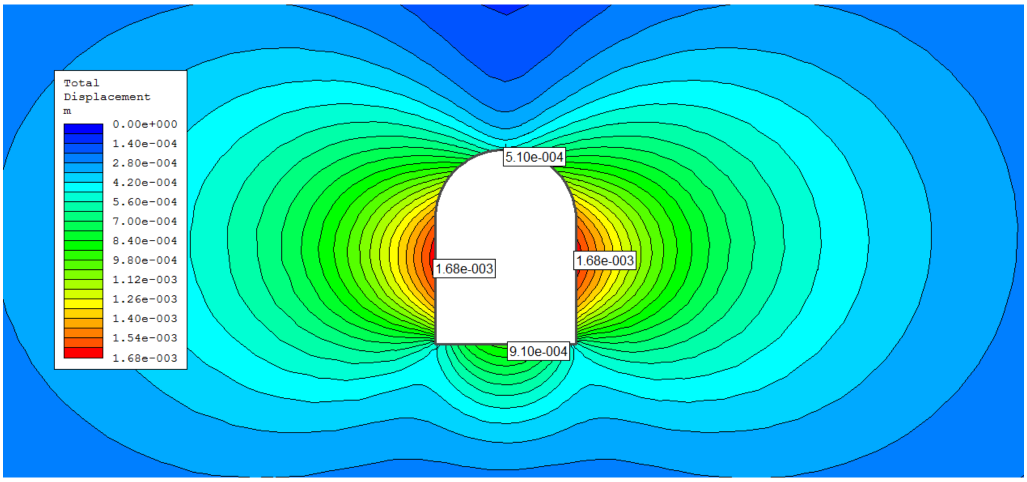


Figure 6.1.1 Plastic zone and total displacement without tunnel support

It can be seen that the maximum displacement is 0.5mm in roof and 0.9mm in invert, and almost no plastic zone. In terms of stability, the plastic fiber shotcrete may be required when necessary.

6.2 Rock Class II

For rock class II, according to LDP curve and Characteristic curve presented in section 2, it is determined to choose ***9m*** as the max. unsupported distance at support installation. The displacement without tunnel support, the displacement after support installation, plastic zone and axial force of rock dowels, the safety factor of plastic fiber shotcrete/ steel rib (including shear and moment) are shown as following.

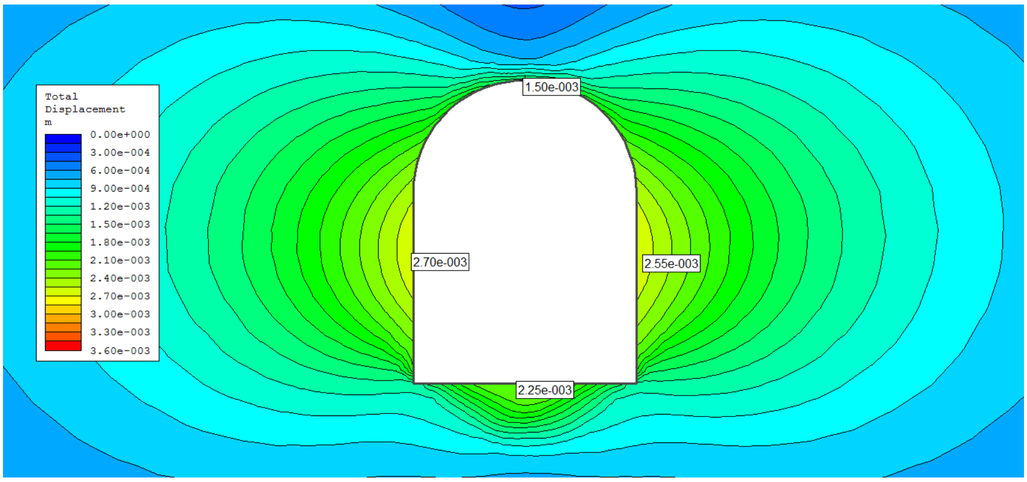


Figure 6.2.1 Total displacement without tunnel support

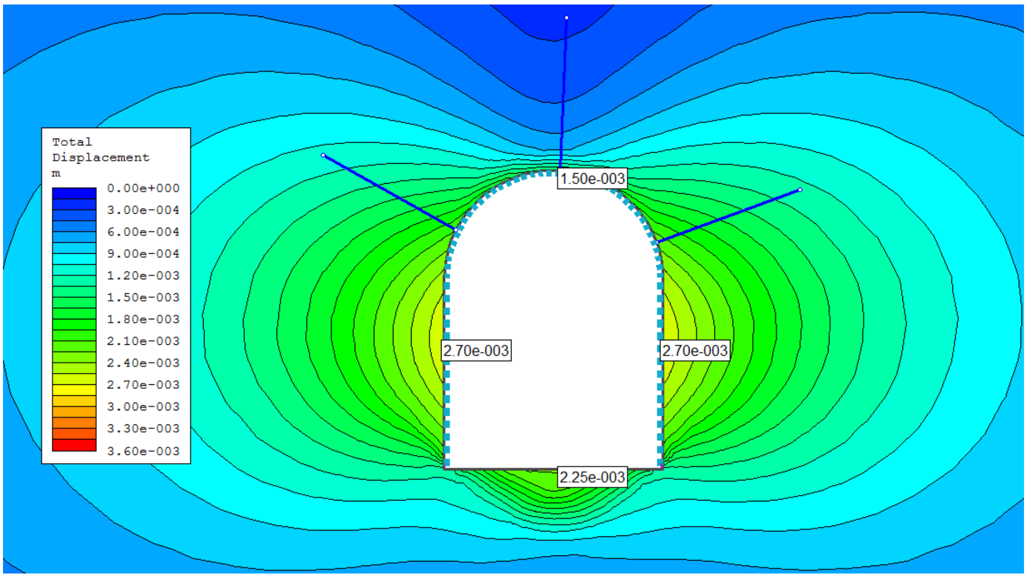


Figure 6.2.2 Total displacement after support installation

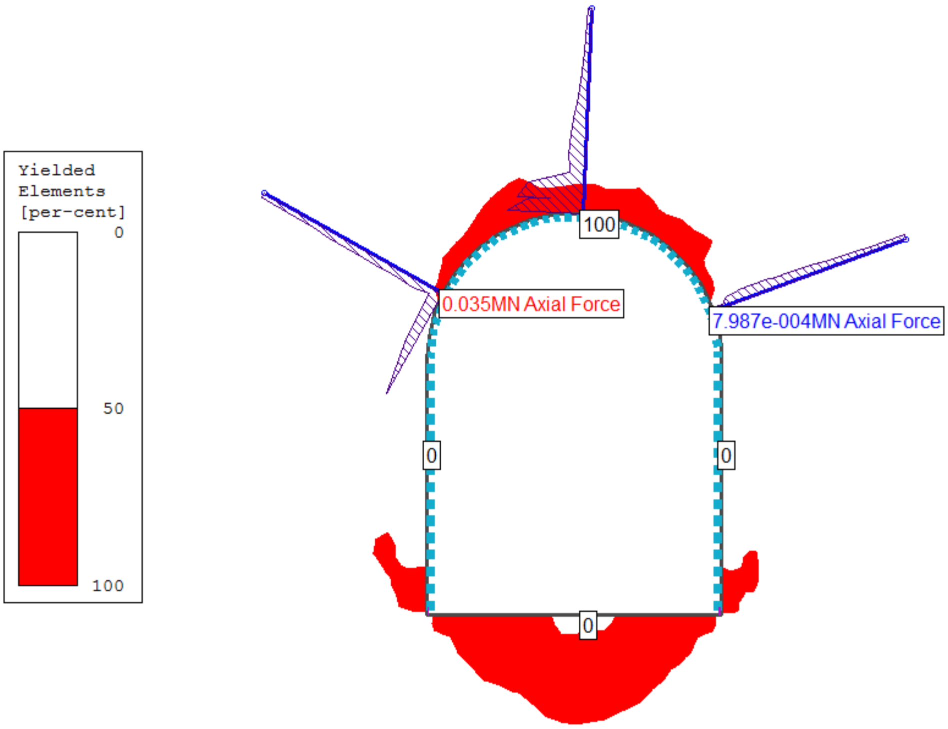


Figure 6.2.3 plastic zone and axial force of rock dowels after support installation

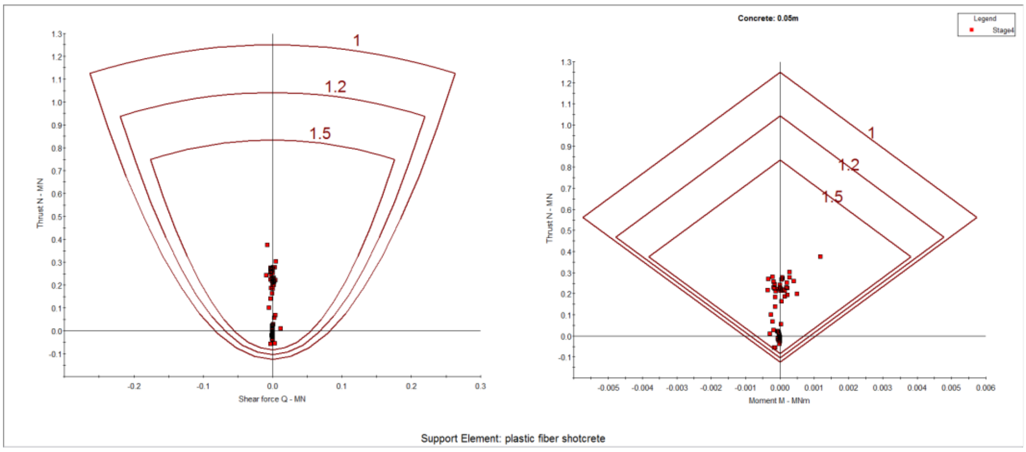


Figure 6.2.4 the safety factor of plastic fiber shotcrete

6.3 Rock Class III

For rock class III, according to LDP curve and Characteristic curve presented in section 2, it is determined to choose ***5m*** as the max. unsupported distance at support installation. The displacement without tunnel support, the displacement after support installation, plastic zone and axial force of rock dowels, the safety factor of plastic fiber shotcrete/ steel rib (including shear and moment) are shown as following.

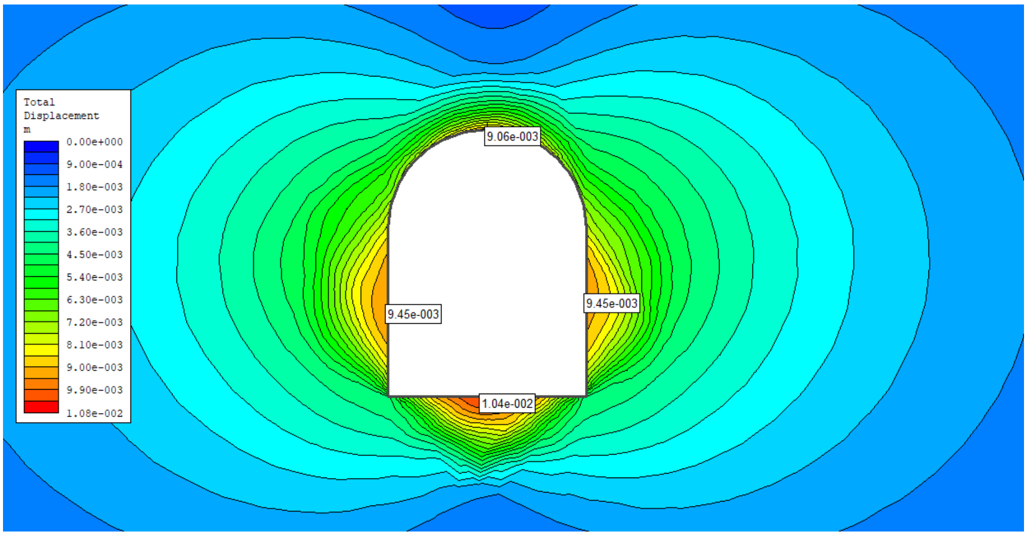


Figure 6.3.1 Total displacement without tunnel support

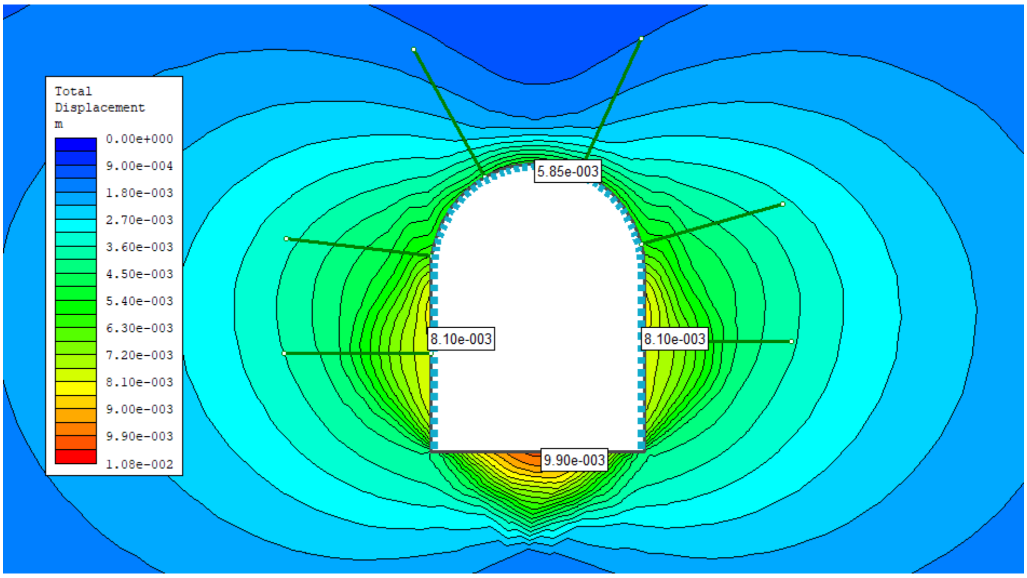


Figure 6.3.2 Total displacement after support installation

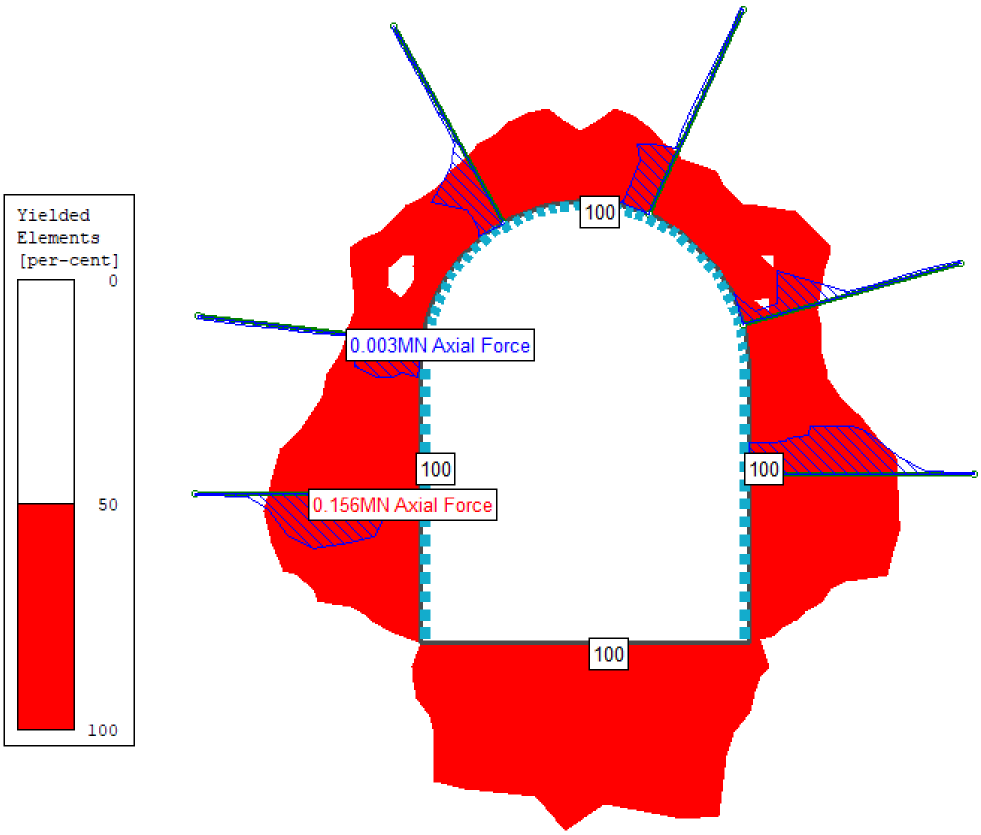


Figure 6.3.3 plastic zone and axial force of rock dowels after support installation

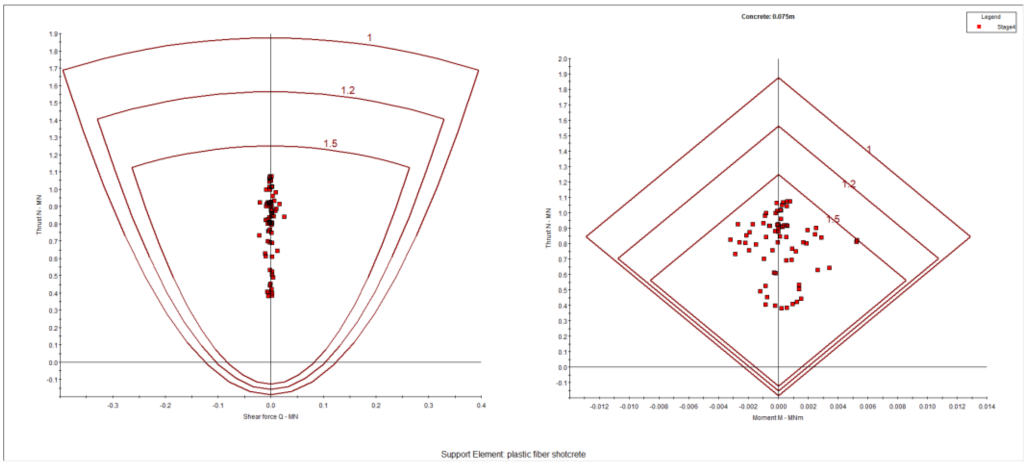


Figure 6.3.4 the safety factor of plastic fiber shotcrete

6.4 Rock Class IV

For rock class IV, according to LDP curve and Characteristic curve presented in section 2, it is determined to choose ***3m*** as the max. unsupported distance at support installation.

Some support system is considered:

(1) The shotcrete thickness is increased to 0.12m;

The displacement without tunnel support, the displacement after support installation, plastic zone and axial force of rock dowels are shown as following.

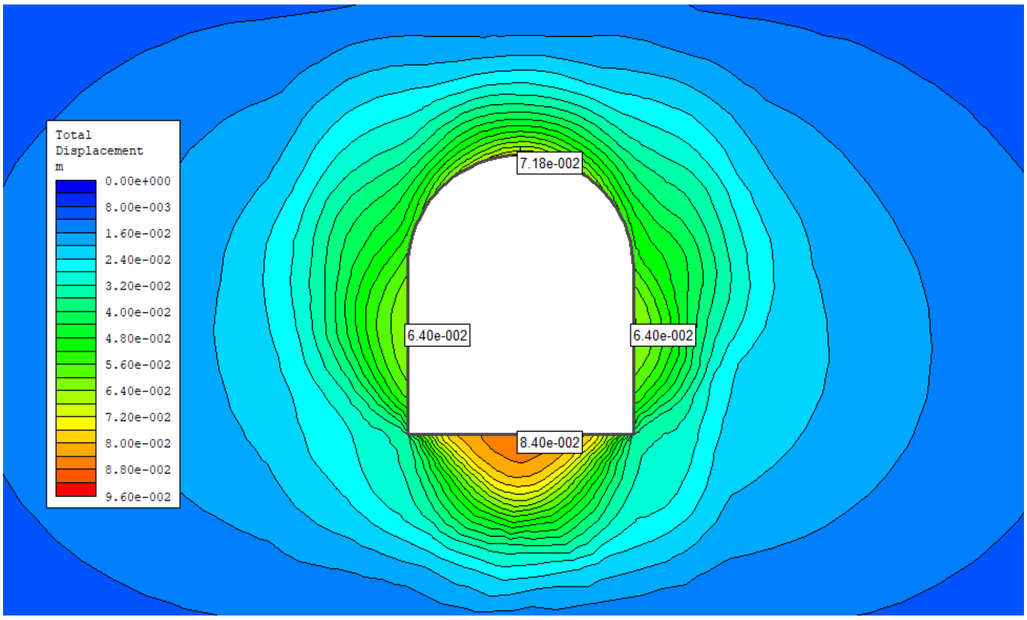


Figure 6.4.1 Total displacement without tunnel support

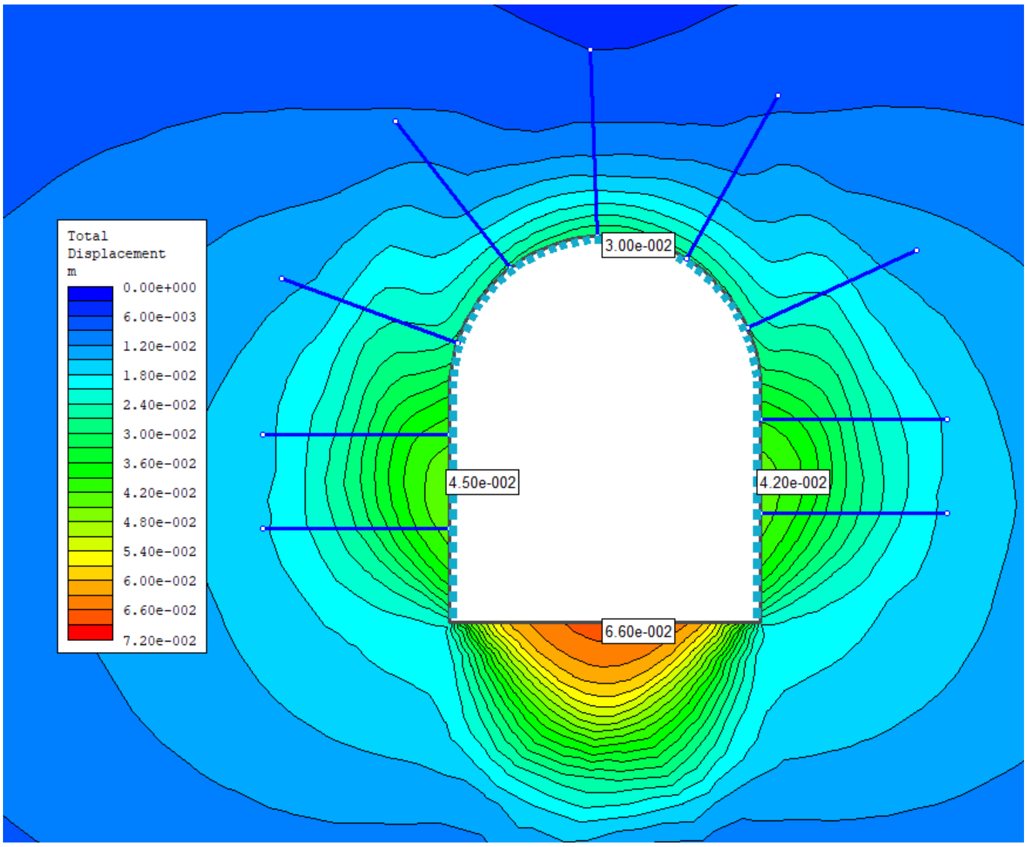


Figure 6.4.2 Total displacement after support installation

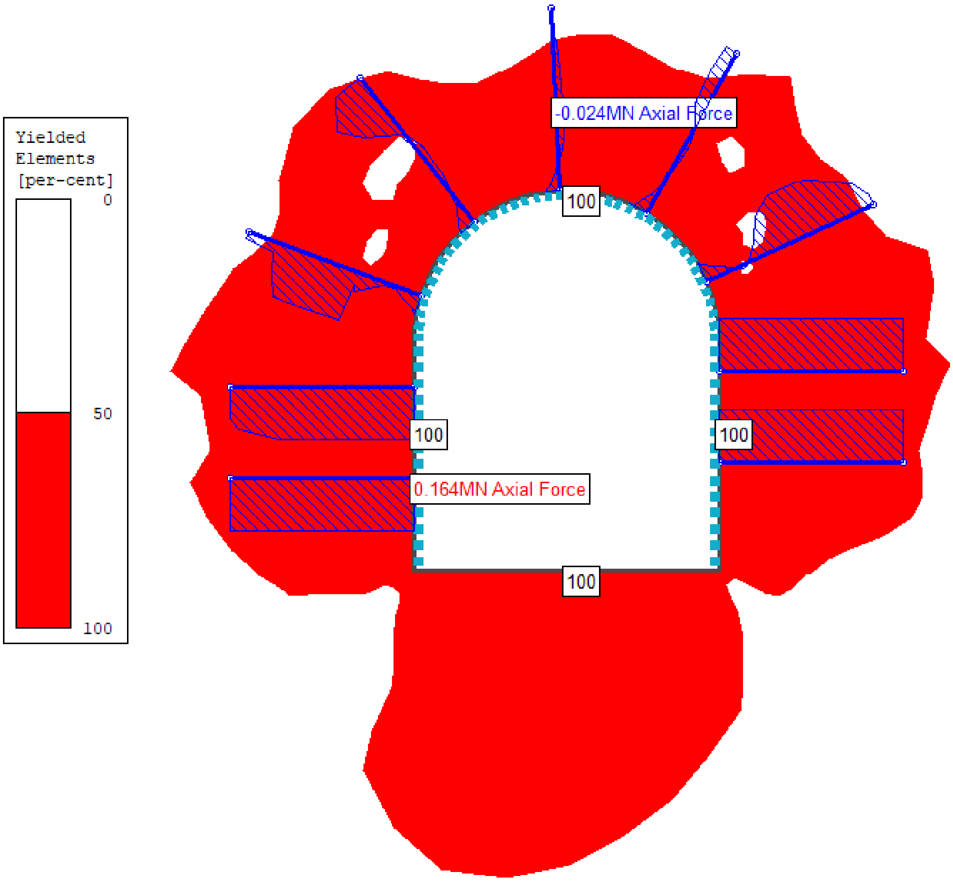


Figure 6.4.3 plastic zone and axial force of rock dowels after support installation

7 Empirical support categories

7.1 Excavation support ratio (ESR)

According to the handbook of the Q-system, a low ESR value indicates the need for a high level of safety while higher ESR values indicate that a lower level of safety will be acceptable. Requirements and building traditions in each country may lead to other ESR-values than those given in table 7.1.

Table 7.1 ESR-values

7	Type of excavation	ESR
A	Temporary mine openings, etc.	*ca.* 3-5
B	Vertical shafts*: i) circular sections ii) rectangular/square section * Dependant of purpose. May be lower than given values.	*ca.* 2.5 *ca.* 2.0
C	Permanent mine openings, water tunnels for hydro power (exclude high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings.	1.6
D	Minor road and railway tunnels, surge chambers, access tunnels, sewage tunnels, etc.	1.3
E	Power houses, storage rooms, water treatment plants, major road and railway tunnels, civil defence chambers, portals, intersections, etc.	1.0
F	Underground nuclear power stations, railways stations, sports and public facilitates, factories, etc.	0.8
G	Very important caverns and underground openings with a long lifetime, ≈ 100 years, or without access for maintenance.	0.5

As for ***construction adit tunnel***, the ***ESR=1.3***.

7.2 Excavation support categories

According to rock parameters for tunnels, rock class I Q>40, rock class II Q>10 & Q<40, rock class III Q>4 & Q<10, rock class IV Q>1 & Q<4.

The span of tunnel is about 5m.

The equivalent dimension=Span or height in meter/ESR=5/1.3≒4m.

As for class I~IV, the empirical supports are shown as follows.

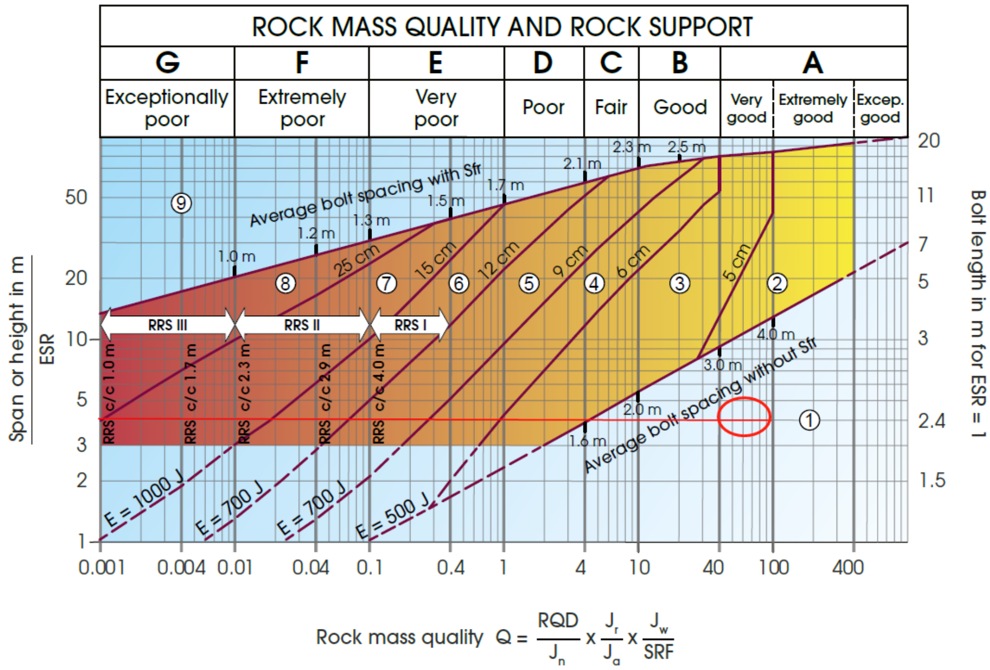


Figure 7.1 Empirical support for rock class I

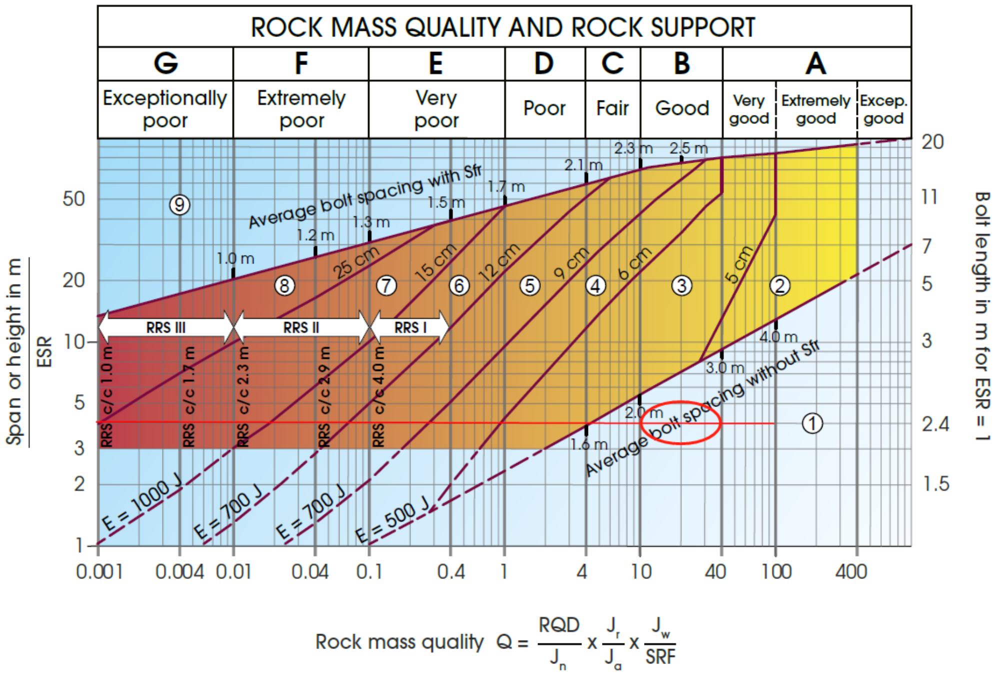


Figure 7.2 Empirical support for rock class II

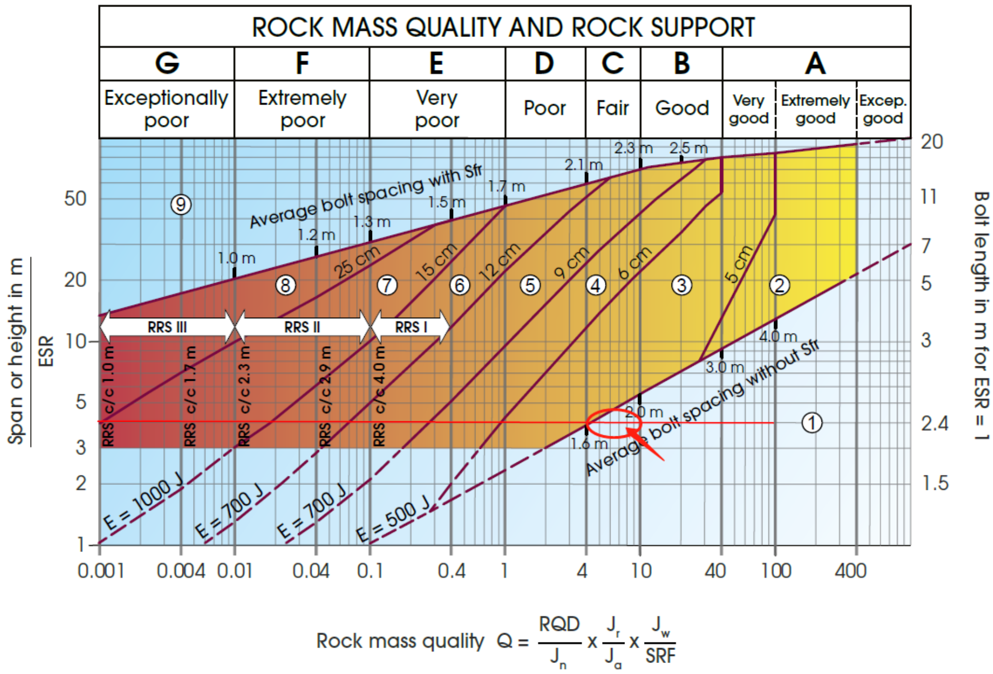


Figure 7.3 Empirical support for rock class III

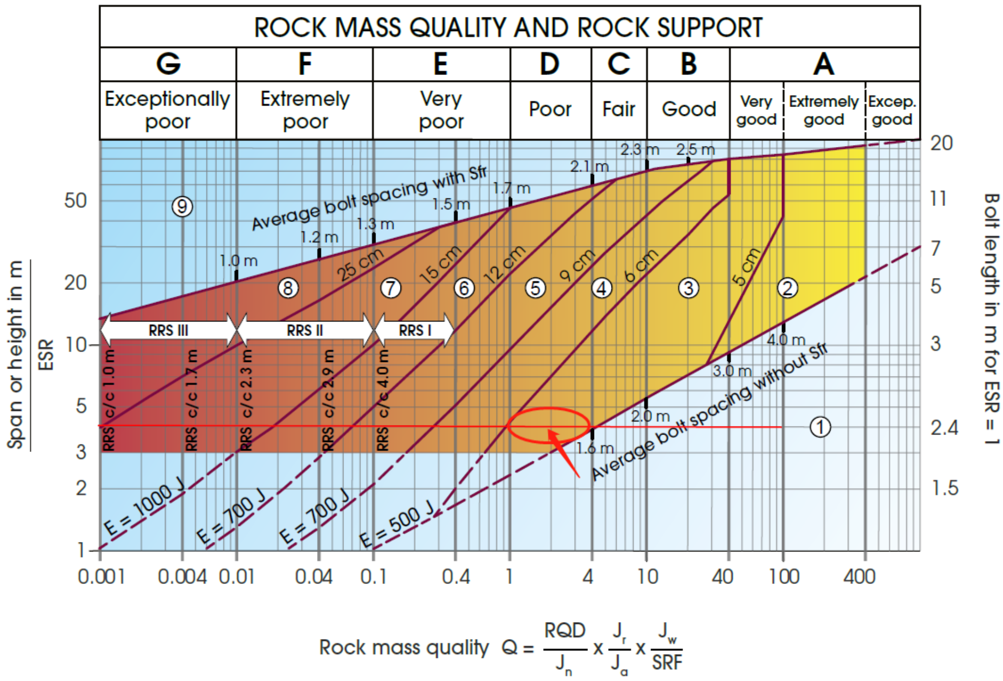


Figure 7.4 Empirical support for rock class IV

Table 7.2 Empirical proposed supports

Item	Unit	Empirical proposed supports			
		I	II	III	IV
Shotcrete thickness	mm	spot	spot	50	60
Rock dowel length	m	spot	spot	2.4	2.4
Rock dowel row spacing	m	spot	spot	1.6~2.0	1.6

8 Wedge Analysis

8.1 Discontinuity Input Data

According to the site investigation, there are three group joints may distribute in the ***adit tunnel***, as is shown in Table 8.1. The stereographic projection of these joints in rock mass in shown in Figure 8.1~8.4. The Mechanical parameter of joints is shown in Table 8.2.

Table 8.1 Occurrence of joints in adit tunnel

Joint NO.	Dip	Dip Direction	Tunnel Direction
J1(foliation)	25°	333°	N29°W
J2	65°	160°	
J3	75°	120°	
J4	60°	65°	

Table 8.2 Mechanical parameter of joints in the adit tunnel

JONIT TYPE	JRC	JCS	C	Φ
		（MPa）	（MPa）	（°）
Foliation(J1)	6	70	0.20	35.0
lithoclasts cemented (J2、J3、J4)	8	50	0.15	28.8

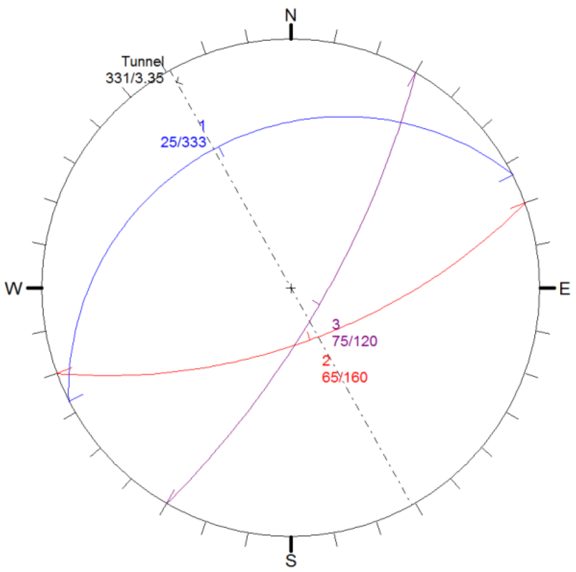


Figure 8.1 Stereographic projection of the joins (J1/J2/J3) in the adit tunnel

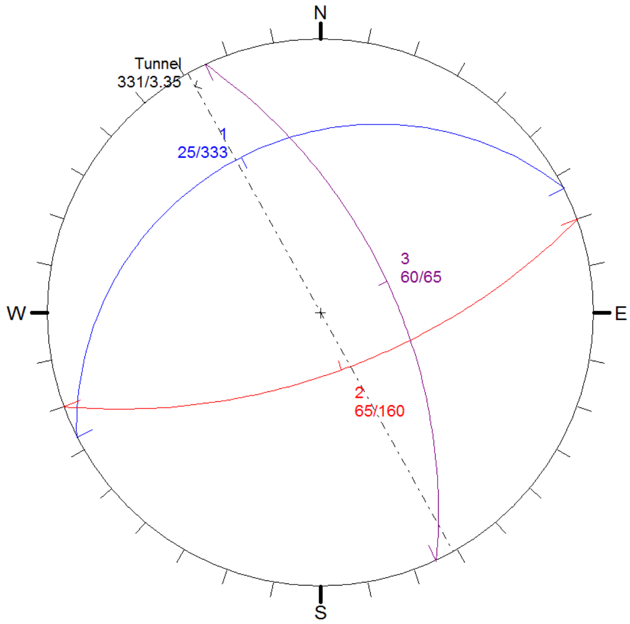


Figure 8.2 Stereographic projection of the joins (J1/J2/J4) in the adit tunnel

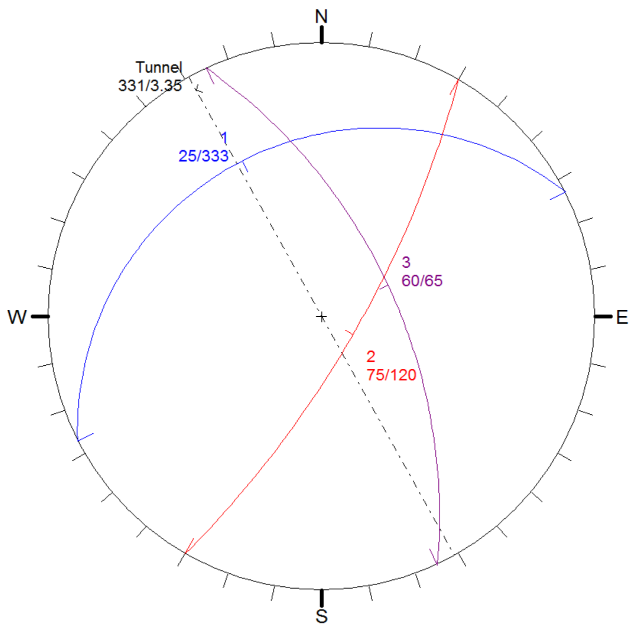


Figure 8.3 Stereographic projection of the joins (J1/J3/J4) in the adit tunnel

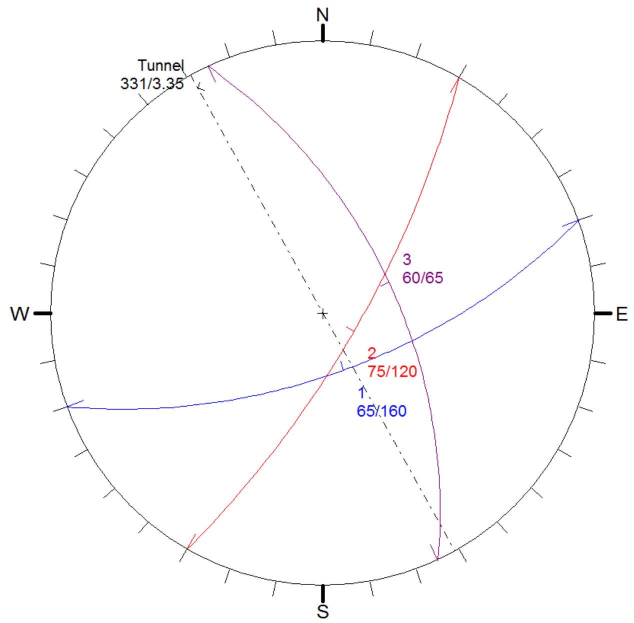
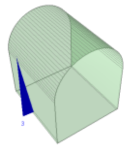
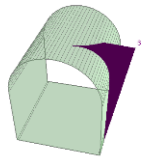
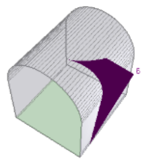
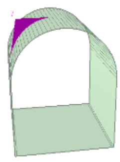
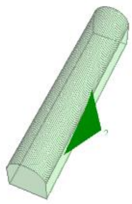
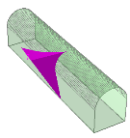


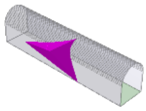
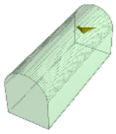
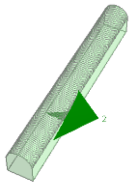
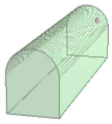
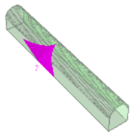
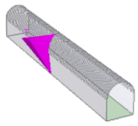
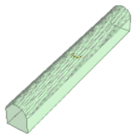
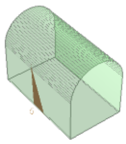
Figure 8.4 Stereographic projection of the joins (J2/J3/J4) in the adit tunnel

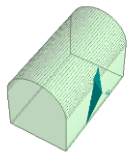
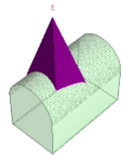
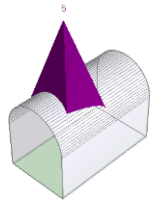
8.2 Wedge Stability Analysis

According to the wedge stability analysis, potential wedges may occur in the excavation periods, which are shown in Table 8.3.

Table 8.3 Instability Block Search Statistics of adit tunnel

S.N.	Fissure set number	Block volume (m^3)	Maximum block height (m)	Safety factor	Development position	Instability mode	Graphical presentation of block
1	J1/J2/J3	0.291	1.63	0.824	Lower left side wall	sliding	
Clean small loose rock blocks						Stable	
2	J1/J2/J3	1.391	2.26	0.532	Upper right side wall	sliding	
After supporting, shotcrete 50mm				10.42		Stable	
3	J1/J2/J3	0.154	1.16	0.058	Crown	fall	
After supporting, shotcrete 50mm				31.54		Stable	
4	J1/J2/J4	12.814	8.92	3.206	Lower right side wall	Stable	
5	J1/J2/J4	38.11	12.26	0.361	Upper left side wall	sliding	

After supporting, shotcrete 50mm				4.156		Stable	
6	J1/J2/J4	0.087	2.51	0	Crown	fall	
Clean small loose rock blocks or shotcrete						Stable	
7	J1/J3/J4	37.209	10.45	89.93	Lower right side wall	Stable	
8	J1/J3/J4	0.1	0.47	0.749	Upper right side wall	sliding	
Clean small loose rock blocks or shotcrete						Stable	
9	J1/J3/J4	69.554	13.01	0.7	Upper left side wall	sliding	
After supporting, shotcrete 50mm				2.99		Stable	
10	J1/J3/J4	0.015	1.98	0	Crown	fall	
11	J2/J3/J4	0.224	1.71	0	Lower left side wall	sliding	
Clean small loose rock blocks						Stable	

12	J2/J3/J4	0.306	1.99	0.195	Upper right side wall	sliding	
Clean small loose rock blocks						Stable	
13	J2/J3/J4	6.895	2.88	0.486	Crown	fall	
After supporting, shotcrete 50mm				4.265		Stable	

It could be seen that, in all the possible joints combination except No.3/5/9/13, the volume of the unstable block is about 0.1~1.3m^3, and the height of the block is about 0.4m~2.5m. In the tunnel, as for class I & class 2, spot shotcrete can restrain the trend of sliding and fall. As for other classes, the pattern bolting can control the sliding and fall. During the excavation process, a reasonable layered excavation sequence shall be used and a reasonable excavation height to avoid large potential unstable blocks.

As for the N0.3/5/9/13 joint combination, after spot and pattern supports, the minimum safety factor of potential wedges with supports in adit tunnel is bigger than the minimum safety factor of 2.0.

9 Conclusions and Recommendations

9.1 Conclusions

As per the calculation fore adit tunnel above, it can be concluded as follows:

(1) For rock class I, it can be seen that the maximum displacement is less than 0.1% of tunnel height (0.001x5600=5.6mm), the systematic support is not required. In the actual excavation process, spot shotcrete may be carried out according to the exposed rock.

(2) For rock class II, minor plastic zone develops at roof, spot rock dowels and 50mm thick shotcrete may be required in the tunnel roof where necessary.

(3) For rock class III, according to geological speculation, the overburden is not greater than 300m, the 75mm thick shotcrete can meet the requirement of tunnel stability.

(4) For rock class IV, according to geological speculation, generally appear near the entrance of the tunnel. If encountered the rock class IV in the location with rapid deformation, stronger/additional support measure such as steel rib will be considered.

(5) For wedge analysis, after shotcrete, the minimum safety factor of potential wedges with supports in adit tunnel is bigger than the minimum safety factor. However, the drainage system in construction periods is extremely important in the stability calculation of wedges, it should be paid more attention in the excavation process.

9.2 Recommendations

Comparing calculated values with empirical values, the recommended supports for each rock type are summarized in the Table 9-1.

Table 9.1 Recommended supports for each rock type

Rock class	Shotcrete thickness (mm)	Rock dowel length (m)	Rock dowel row spacing(m)	Steel support/Lattice girder spacing (m)
I	*spot*	*3.0*	*spot*	*/*
II	*50*	*3.0*	*2.5(roof)*	*/*
III	*75*	*3.0*	*2.0*	*/*
IV	*120*	*3.0*	*1.5*	*spot*